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Doc No. 22

A Report for

Fred Devine Diving & Salvage Company

Geotechnical and Seismic Evaluation

Proposed Expansion, Office and Warehouse Buildings, 6211 N. Ensign, Portland, Oregon

Project EAAX-95-0286
Report 09-075-1442
July 19, 1995

BRAUN INTERTEC NORTHWEST

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INTERTEC

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Engineers and Scientists Serving the Built and Natural Environments

July 19, 1995

Project No. EAAX-95-0286
Report No. 09-075-1442

Mr. J.H. Leitz, President
Fred Devine Diving &
Salvage Company
6211 N. Ensign
Portland, OR 97217

Dear Mr. Leitz:

Re: Geotechnical and Seismic Evaluation for the Proposed Expansion, Office and Warehouse Buildings, 6211 N. Ensign, Portland, Oregon

The geotechnical evaluation you authorized on June 12, 1995, has been completed. The purpose of these services was to assist you, the architect and the engineer in designing foundations and preparing plans and specifications for construction of the new buildings. The evaluation was completed in general accordance with our Proposal No. EAAX-95-P102 and our Confirmation of Authorization for Services to you dated June 13, 1995.

Summary of Results

Three standard penetration test (SPT) borings (B-1 to B-3) extending to depths of 16½ feet to 51½ feet below existing grades were completed in the proposed building areas. The general soil profile was approximately 16½ feet of very loose to medium dense dredged sandy and silty fill (SP, SM) underlain by very loose silt-sand mixtures (SP, SM, ML) extending to a depth of 40 feet. These upper soils were underlain by very soft silts extending to maximum boring termination depth of approximately 51½ feet. Groundwater was encountered at approximately 25 feet below the existing ground surface during our exploration. Seasonal high groundwater level is estimated at 10 feet to 15 feet below the lowest site grade.

Maximum shear moduli (G_{max}) were calculated based on SPT N-values using the procedure described in Earthquake Engineering Research Center Publication No. UCB/EERC-84/14, September, 1984. The G_{max} values ranged from 750 ksf to 2,100 ksf for the upper 50 feet of soils at the site.

Our liquefaction analyses were performed using the Seed et. al. SPT procedure for soils underlying the site. A low factor of safety against liquefaction was noted at depths ranging from 10 feet to 13 feet and 30 feet to 40 feet below the existing ground surface, thus, indicating moderate liquefaction potential at the site during a seismic event.

Summary of Recommendations

Based on the results of the SPT borings, it is our opinion that the proposed warehouse expansion and the office building may be supported on conventional shallow spread footings provided liquefaction hazards are not considered.

Shallow Foundations

Due to the presence of very loose to loose dredged sandy and silty fill soils present at depths of 5 feet to 12.5 feet below the existing office building, we recommend that the footing excavations for the proposed office expansion be surficially compacted using a hand held plate compactor prior to the placement of concrete. In addition, we recommend the use of negative reinforcement for the footings as a crack control measure.

Shallow spread footings for the warehouse and office expansions can be proportioned using the following bearing pressure criteria.

- Net maximum allowable bearing pressure: 1,500 psf
- Increase bearing capacity by 1/3 for total loads, including seismic and wind loading.

For a shallow foundation system designed in accordance with our recommendations, we anticipate a maximum total settlement of 1 inch and differential settlement of 1/2 inch.

The moderate potential for displacement due to liquefaction will remain if the proposed structures are supported on a shallow foundation. If it is desired to minimize the potential for liquefaction and associated hazards, then a deep foundation system may be considered for supporting the proposed office building. We should be contacted to provide specific recommendations for such a deep foundation system.

Floor Slabs

Slab-on-grade for the warehouse and office expansion can be designed using a modulus of vertical subgrade reaction value of 200 pci. Slab-on-grade should be underlain by a minimum of 6 inches of free-draining well-graded gravel or crushed rock base course.

Seismic Considerations

Subsurface conditions at the site can be modelled by the Uniform Building Code (UBC) Soil Profile Type "S_3" with "S" factor of 1.5. Structural analysis based on the UBC equivalent static design method would be appropriate for this project.

General

It is our understanding that the existing office building will be lifted and temporarily supported above ground for the construction of a ground floor at or near existing grades. Our services should be retained to provide recommendations for a temporary support system during construction.

Please refer to the attached report for a description of our analyses and recommendations. If we can provide additional assistance, or observation and testing services during design and construction, please call (503) 289-1778 or (800) 783-6985.

Sincerely,

Sudhir Adettiwar
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
5944
OREGON
JULY 14 1967
CHARLES R. LANE

Charles R. Lane, P.E
Senior Engineer

sa:crl/pas

Attachments: Geotechnical Evaluation Report

c: Braun Intertec Corporation, St. Cloud

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References

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BRAUN
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Engineers and Scientists Serving the Built and Natural Environments

July 18, 1995

Project EAAX-95-0286
Report 09-075-1442

Geotechnical Evaluation
Proposed Expansion, Office and Warehouse Buildings
611 N. Ensign
Portland, Oregon

1.0 Introduction

At your request, we have conducted a geotechnical evaluation for the subject project in general accordance with the scope of work as outlined in our proposal to Fred Devine & Salvage Company dated June 13, 1995. Authorization for our services was provided by Mr. J.H. Leitz, the president of Fred Devine and Salvage Company.

2.0 Project Description

The project site is located on the bank of Swan Island basin at 6211 N. Ensign in Portland, Oregon as shown in the Site Location Map, Figure 1, attached. It is occupied by a two-story office building and an L-shaped metal warehouse building as shown in the Schematic Site Plan, Figure 2, attached.

We understand that present plans are to expend the existing warehouse building in the northwestern portion. The office building will be elevated by approximately 5½ feet and supported temporarily above ground for the construction of an additional ground floor at an elevation of El. 31.0 above Mean Sea Level (MSL).

3.0 Purpose and Scope

The purpose of our evaluation was to assess the subsurface soil conditions at the site in order to provide appropriate recommendations for site preparation and foundation design. In general, our evaluation included the following authorized scope of work items:

3.1 Subsurface Exploration

In order to ascertain soil conditions at the site, three Standard Penetration Test (SPT) borings (B-1 to B-3) extending to depths of 16½ feet to 51½ feet were made using a truck-mounted hollow stem drilling auger in general accordance with ASTM D-1586 procedure. Boring locations are shown in Figure 2 attached. SPT Samples were taken at 2.5 foot intervals in the first 15 feet followed by 5 foot intervals.

The SPT drilling was performed by driving a 2-inch O.D. split-spoon sampler into the undisturbed formation at the bottom of the boring with repeated blows of a 140-pound pin-guided hammer falling 30 inches. The number of blows required to drive the sampler 1 foot was a measure of the soil consistency. Samples were identified in the field, placed in sealed containers and transported to the laboratory for further classification and testing. Results of all the SPT borings (Log of Borings) are included in Appendix A.

3.2 Laboratory Evaluation

Selected samples of the subsurface soils were returned to our laboratory for further evaluation to aid in classification of the materials and to help assess their strength and compressibility characteristics. The laboratory evaluation consisted of visual and textural examinations, moisture content determinations, gradation analysis and Atterberg limits determinations.

Laboratory testing was not deemed necessary for this project.

3.3 Engineering Analyses

Engineering analyses were performed using the results of subsurface and laboratory investigations. Our analyses included bearing capacity calculations, settlement estimation, and seismic hazard evaluations. In addition, recommendations were developed addressing general site preparation procedures, excavation/slopes, floor slabs, retaining walls, drainage, and pavements. Results of engineering analyses and our recommendations are discussed in Chapter 6 of this report.

4.0 Surface and Subsurface Features

Surface and subsurface features at the site described below were present at the time of our field explorations.

4.1 Site Description

The project site is a rectangularly shaped, partially developed parcel of land located on the bank of Swan Island Basin at 6211 Ensign in Portland, Oregon. The site is occupied by a two-story office building, an L-shaped warehouse, and a mooring dock.

A review of a U.S.G.S. topographic map indicated site grade elevations around El. 33 above MSL. Based on our knowledge of the project area, we believe that dredged sandy fill and silty fill was placed on the site to achieve existing grades during original earthwork operations.

4.2 Soils and Geology

The project area is underlain by approximately 16.5 feet of dredged sandy fill and silty fill soils placed by humans. These most recent fill soils are underlain by Quaternary and Pleistocene Age alluvial deposits of the Willamette River. These alluvial deposits generally consist of sands, silts, gravels, and sand-silt mixtures and extend to depths of 150 feet to 250 feet. The near surface deposits overlie the discontinuous gravel layers of the Troutdale Formation extending to a depth of approximately 400 feet to 450 feet. Below these depths, Columbia River basalt and other volcanic rocks which constitute the local bedrock for the project area are present.

Specific soil units encountered during our exploration are described as follows.

> <u>Dredged Sandy Fill and Silty Fill</u> - Very loose to medium dense and fine to medium grained sands to silty sands (SP, SM) are present in the upper 16.5 feet. An old gravel pavement was encountered at 12½feet to 15 feet in boring B-1.
>
> <u>Alluvial Units</u> - Upper fill soils are underlain by very loose to medium dense sand-silt mixtures (SP, SM, ML) extending to a depth of 40 feet. Below 40 feet, very soft silt (ML) layers that extend to a maximum boring termination depth of 51½ feet are present.

4.3 Groundwater

Groundwater was encountered at 25 feet below existing grades in boring B-1 during our exploration.

Variations in groundwater levels should be expected seasonally, annually and from location to location. We anticipate that the groundwater table may rise during months of peak runoff. The Soil Conservation Survey (SCS) book on Multnomah County classifies near surface soils at the site as "Urban Land, 0 to 3 percent slopes". According to the SCS book, a seasonal high groundwater table for these soils is within approximately 1 foot of the original ground surface, i.e. approximately 15 feet below existing ground surface. A review of the site plan prepared by Lee Engineering, Inc., in 1984 during the construction of a mooring dock at the site also indicated a seasonal high groundwater table within 10 feet to 15 feet of existing ground surface.

4.4 Shear Moduli Estimation

Maximum shear moduli (G_{max}) for subsurface soils were calculated from the SPT boring data using a procedure described in Earthquake Engineering Research Center publication UCB/EERC-84/14 (Reference 1). G_{max} values ranged from 750 ksf to 2,100 ksf for the upper 50 feet of soils at the site. Based on our knowledge of the geology of the site vicinity and pile driving data obtained from the mooring dock project file, we estimated G_{max} values for soils below 50 feet. A G_{max} profile for the site is shown in Figure 3, attached.

5.0 Seismic Considerations

The site is located within Seismic Zone 3 with a seismic zone factor of 0.3 as indicated by Figure 16-1 and Table 16-I of the Uniform Building Code (UBC) of 1994. Based on our subsurface exploration, it is our opinion that the soil profile at the site is S3 (Type 3).

The site was evaluated for the effects of seismic hazards in accordance with the requirements of Section 2905 of the Oregon Structural Specialty Code (OSSC). Based on our evaluation, we believe that the potential for ground rupture, subsidence, landslides, tsunami flooding and seiche at the site is minimal. Our evaluation of the potential for liquefaction and ground shaking are discussed in the following paragraphs.

5.1 Liquefaction

Liquefaction occurs when a saturated deposit of loose, fine grained sands is subjected to strong earthquake shaking. Loose saturated sand deposit will have a tendency to compact and thus decrease in volume. If this deposit is saturated and, thus, cannot drain rapidly, there will be an increase in the pore water pressure. With increasing oscillation, the pore water pressure can increase to the value of the overburden pressure. The shear strength of a cohesionless soil is directly proportional to the effective stress which is equal to the difference between the overburden pressure and the pore water pressure. Therefore, when the pore water pressure increases to the value of overburden pressure, the shear strength of the soil reduces to zero and the soil deposit turns into a liquefied state.

Liquefaction potential of the soils at the site was evaluated using the Seed, et al. SPT procedure (Reference 2). Seismic events for the site were postulated in accordance with OSSC. Cyclic stress ratios causing liquefaction during postulated seismic events were calculated at various depths and were compared with the liquefaction resistance stress ratios for computing a factor of safety against liquefaction. Our analyses indicated a low factor of safety values for soils at depths of 10 feet to 13 feet, and 30 feet to 40 feet, thus indicating moderate liquefaction potential at the site.

5.2 Ground Response Analyses

Postulated seismic events were simulated in accordance with OSSC to represent input ground motion. Preliminary ground response (shaking) analyses were performed using the software program SHAKE 91 (Reference 3) for the postulated seismic events. Dynamic soil input parameters included maximum shear moduli (Figure 3), modulus reduction relationships (Figure 4 and 5), and damping ratio relationships (Figure 4 and 5). These input parameters were obtained from available soil dynamics literature (References 1 and 4).

Peak Ground Acceleration (PGA) values were determined from the results of SHAKE 91 analyses and attenuation relationships published by Boore et al. and Youngs et al. (References 5 and 6). Based on the results of our preliminary analyses, it is our opinion that the lateral earthquake loads calculated using the Uniform Building Code (UBC) equivalent static design methods and the S_3 soil profile would be appropriate for this project.

6.0 Conclusions and Recommendations

Based on the results of our field work, laboratory evaluation and engineering analysis, it is our opinion that the site is suitable for the proposed structure and associated improvements provided the following recommendations are incorporated into the design and construction of the project.

6.1 Site Preparation

In general, we recommend that all structural improvement areas be drained of surface water (pumping from a sump hole, if necessary), and stripped of surface vegetation, topsoil materials, highly saturated disturbed soil, and any other deleterious materials encountered at the time of construction.

We envision that initial site preparation will consist of grading operations. Prior to the placement of any fills during grading operations at the site, all exposed subgrade surfaces should be proofrolled with a half-loaded dump truck. Areas found to be soft or otherwise unsuitable for support of structural loads should be overexcavated and replaced with compacted fill. We believe that on-site soils in the upper 25 feet can be used as structural fill provided soils are free from organic materials and debris.

Selected samples of the materials to be used for fill should be submitted to our laboratory in order to evaluate the maximum dry density and optimum moisture content of the material and to evaluate the suitability of the soil for use as fill. All required structural fill materials placed in the building and pavement areas should be moistened or dried as necessary to near optimum moisture conditions and compacted by mechanical means to a minimum of 95 percent of the maximum dry density as determined by the modified Proctor test (ASTM D-1557). Fill materials should be placed in layers that do not exceed about 8 inches when compacted.

6.2 Excavations and Construction Dewatering

Deep excavations are not anticipated for this project. In general, all excavations at the site associated with confined spaces must be completed in accordance with local, state, or federal regulations. Soils at the site in the upper layers can be classified as Type C soils in accordance with the current Occupational Safety and Health Administration (OSHA) guidelines. Consequently, Excavations deeper than 5 feet and up to 10 feet may be made with side slopes laid back at a minimum inclination of 33^0 (1.5 H : 1.0 V). For excavations deeper than 10 feet, we should be contacted to provide sloping or benching recommendations.

Groundwater seepage should be anticipated in excavations during the wet season. For excavations shallower than 5 feet, pumping from sumps outside the limits of the excavation may control groundwater seepage.

Our recommendations for excavations/slopes and dewatering are provided only for the benefit of the contractor and other parties involved in the project. It should be noted that job site safety is the complete responsibility of the project contractor.

6.3 Shallow Foundations

The conventional shallow spread footings can be designed for a net maximum allowable bearing pressure of 1,500 psf. This value can be increased by 1/3 for the total of all loads that may include short term wind or seismic loads. Footings should have a minimum dimension of 18 inches and be placed at least 18 inches below the finished exterior grades to minimize the potential for a localized shear failure and for frost protection.

Due to the presence of very loose to loose dredged sandy fill soils present at depths of 5 feet to 12½ feet below the existing office building, we recommend that the footing excavations for the proposed office expansion be surficially compacted using a hand held plate compactor prior to the placement of concrete. In addition, we recommend the use of negative reinforcement for the footings as a crack control measure.

Allowable lateral frictional resistance between the base of footings and the granular subgrade can be expressed as the applied vertical load multiplied by a coefficient of friction of 0.30. In addition, lateral loads may be resisted by passive earth pressures based on an equivalent fluid density of 250 pounds per cubic foot (pcf) on footings poured "neat" against insitu soils or properly backfilled with structural fill. This recommended value includes a factor of safety of approximately 1.5, which is appropriate due to the amount of movement required to develop full passive resistance.

We estimate that foundations designed and constructed in accordance with the above recommendations will experience total settlements generally less than 1-inch and differential settlement between columns generally less than ½-inch.

A moderate potential for liquefaction exists at the site. If it is desired to minimize the liquefaction potential, then a deep foundation system consisting of piles may be considered for supporting the office building. Additional investigation will be required for developing recommendations for a deep foundation system.

6.5 Floor Slab Support

For any proposed floor slab-on-grade , we recommend that floor slabs be underlain by a minimum of 6 inches of free-draining (a maximum size of 3/4 inch with less than 5 percent passing the No. 200 sieve) well-graded gravel or crushed rock base course. The base course material should be compacted to at least 95 percent of the maximum dry density obtainable by the ASTM D 1557 test procedure. A modulus of vertical subgrade reaction value, K, of 200 pounds per square inch per inch of settlement (pci) may be used for slab-on-grade thickness design using plate load test analogy.

The crushed rock should provide a capillary break to limit migration of moisture through the slab. If additional protection against moisture vapor is desired, a vapor retarding membrane may also be incorporated into the design. Factors such as cost, special considerations for construction, and the floor coverings suggest that decisions on the use of vapor retarding membranes be made by the architect and owner.

6.6 Retaining Walls

All backfill for retaining walls, foundation walls, etc., should be select granular material (sand and/or sandy gravel). We anticipate that onsite material in the upper 25 feet will be suitable for this purpose. All backfill behind walls should be placed in lifts not exceeding 8 inches in final thickness that are compacted to at least 90 percent of the maximum dry density obtainable by the ASTM D 1557 test procedure. Care in the placement of fill behind walls must be taken in order to insure that undue lateral loads are not placed on the wall. A well-designed drainage system should be provided behind the walls to control undue hydrostatic pressures.

Lateral earth pressures on walls which are not restrained at the top, such as retaining walls, etc., may be calculated on the basis of an equivalent fluid pressure of 35 pcf for level backfill and 60 pcf for steeply sloping backfill. Walls that are restrained from yielding at the top may be calculated on the basis of an equivalent fluid pressure of 55 pcf for level backfill and 90 pcf for steeply sloping backfill. Lateral loads may be resisted by passive pressures acting against footings and by frictional resistance between foundation elements and supporting soils. An equivalent fluid density of 250 pounds per cubic foot (pcf) and a friction factor of 0.30 may be used for design for foundations bearing on and resisted by native soils. The recommended equivalent fluid density includes a factor of safety of 1.5 which is appropriate due to the amount of movement required to develop full passive resistance.

6.7 Drainage Considerations

In general, any areas of the building which are to be developed below the exterior site grade must be provided with a well-designed drainage system in order to control hydrostatic pressures against walls, seepage of groundwater through base walls, etc. Surface run-off from roofs, parking areas, etc., should be tightlined to the storm sewer or other approved disposal areas. All pavement and parking areas should be sloped away from the building to prevent ponding of water near the buildings.

6.8 Pavement Recommendations

The following recommendations are presented as preliminary for your consideration. The civil engineer for the project may have more traffic and project design data available than is presently known and may wish to modify and refine these pavement sections. We will, upon request be pleased to provide a more detailed pavement design when definite traffic and building plans are available.

Prior to placing the base or leveling course the subgrade should be proof-rolled with a loaded dump truck to detect areas or pockets of unusually soft material. These should then be excavated and replaced with suitable compacted fill.

6.8.1 Asphalt Pavement

We recommend that asphalt pavement be designed for an assumed subgrade California Bearing Ratio (CBR) of 10. A typical asphalt pavement section would be:

	Thickness	
	Entrance Service Roads	Car Parking
Asphalt Pavement (Ore. St. Class C)	4 inches	2½ inches
Crushed Rock Base (Ore. St. Spec.)	12 inches	8 inches

Asphalt pavement base course materials should consist of well-graded 1½-inch or ¾-inch minus crushed rock, having less than 5 percent material passing the No. 200 sieve. The base course and asphaltic concrete materials should conform to the requirement set forth in the latest edition of the State of Oregon, Standard Specifications for Highway Construction. The base course material should be compacted to at least 95 percent of the maximum density as determined by the ASTM D-1557 test designation. The asphaltic concrete material should be compacted to at least 90 percent of the theoretical maximum density as determined by ASTM D-2041 (Rice Specific Gravity).

6.8.2 Concrete Pavement

We recommend that concrete pavement be designed for a modulus of subgrade reaction of 200 pci. A typical concrete pavement section would be:

	Thickness	
	Entrance Service Roads	Car Parking
Concrete (4,000 psi)	7.5 inches	4 inches
Leveling Coarse (Sand or All-Weather Base)	2 inches	2 inches

6.9 Construction Monitoring

We request that we examine and identify all soil exposures created during project excavations in order to verify that soil conditions and bearing pressures are as anticipated. We recommend that the structural fills be continuously observed and tested by our representative in order to evaluate the thoroughness and uniformity of their compaction. If possible, samples of fill materials should be submitted to our laboratory for evaluation prior to placement of fills on the site.

Our firm should be contacted to provide recommendations for a temporary support system for elevating the office building during construction.

Costs for the recommended observations and consulting during construction are beyond the scope of this current consultation. Such future services would be at an additional charge.

7.0 General

The conclusions and recommendations presented in this report are subject to the following general conditions.

7.1 Use of Report

This report is for the exclusive use of the addressee and their representative to use to design the proposed structure described herein and prepare construction documents. The data, analyses and recommendations may not be appropriate for other structures or purposes. We recommend that parties contemplating other structures or purposes contact us. In the absence of our written approval, we make no representation and assume no responsibility to other parties regarding this report.

7.2 Level of Care

Services performed by the geotechnical and materials engineer for this project have been conducted with that level of care and skill ordinarily exercised by members of the profession currently practicing in this area under similar budget and time restraints. No warranty, expressed or implied, is made.

We will be pleased to provide such additional assistance or information as you may require in the balance of the design phase of this project and to aid in construction control or solution of unforeseen conditions which may arise during the construction period.

References

1. Seed et al.; Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils, University of California, EERG, UCB/EERC-84/14, September, 1984.

2. Seed et al.; Liquefaction of Soils During Earthquakes, Geotechnical Special Publication No. 20, pp. 94-104.

3. SHAKE 91; A Computer Program for Conducting Equivalent Linear Seismic Response Analysis of Horizontally Layered Soil Deposits, University of California, Davis, CA, November 1992.

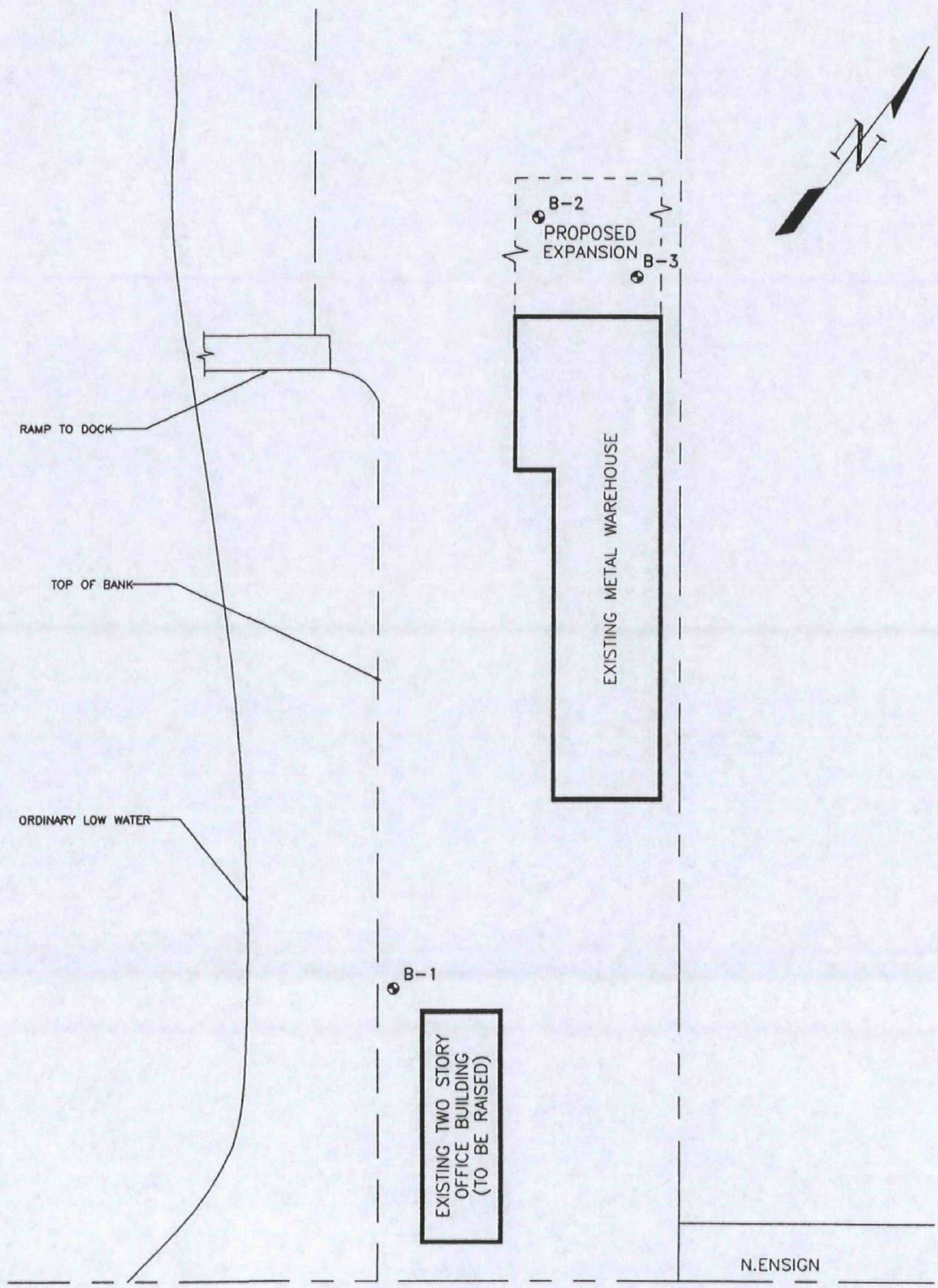
4. Sun et al.; Dynamic Moduli and Damping Ratios for Cohesive Soils, University of California, EERC, UCB/EERC-88/15, August 1988.

5. Boore, et al.; Estimation of Response Spectra and Peak Accelerations from Western north America Earthquakes: An Interim Report, USGS Open File Report 93-509.

6. Youngs, et al.; Near Field Ground Motion on Rock for Large Subduction Earthquakes, Geotechnical Special Publication No. 20, pp. 445-460.

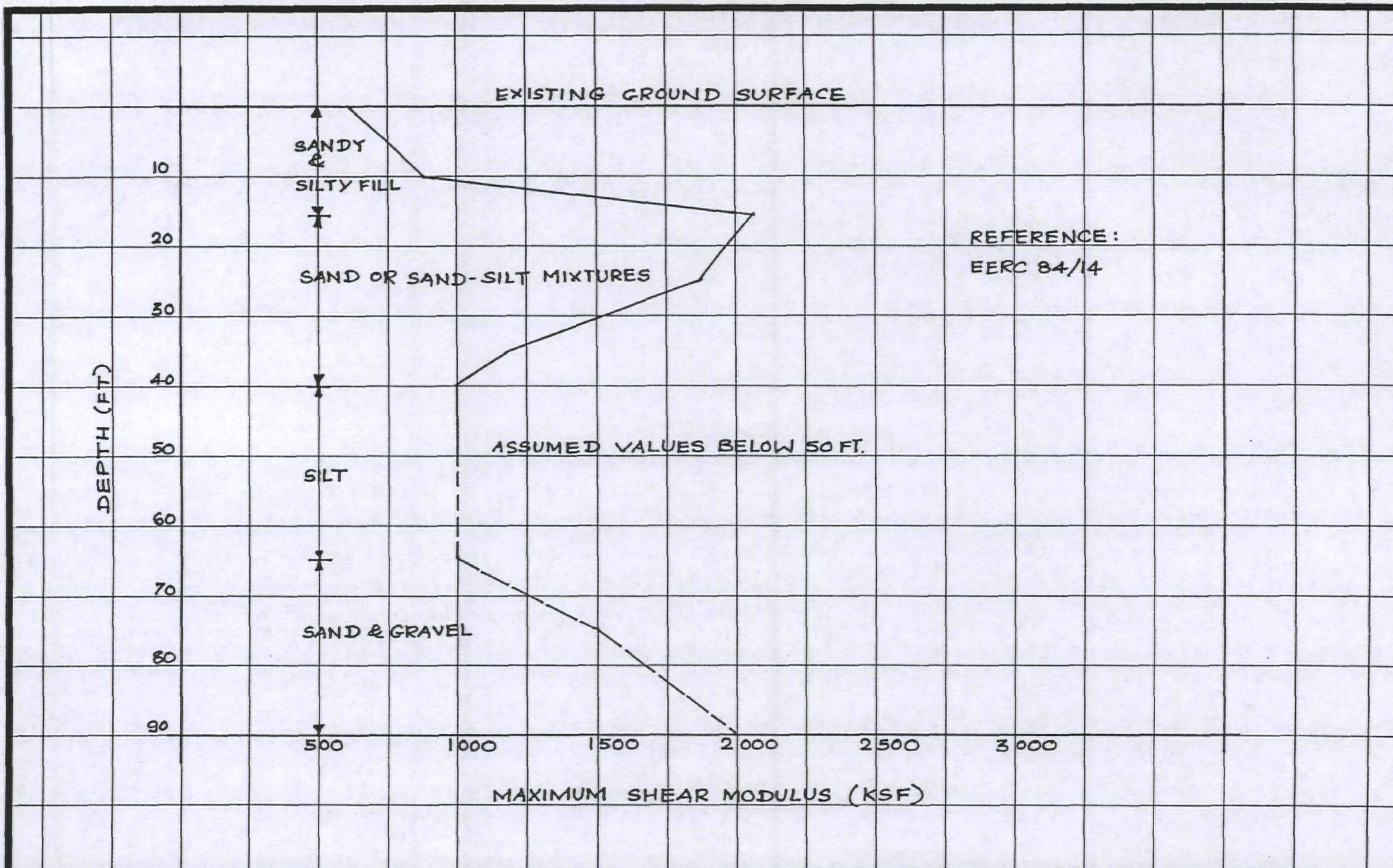
Figures

BRAUN INTERTEC	SITE LOCATION MAP FRED DEVINE DIVING & SALVAGE 6211 N. ENSIGN PORTLAND, OREGON	DRAWN BY: R.F. JOB NO: EAAX-95-0286 DATE: 7-18-95 DRAWING NO: 1 OF 1 FIGURE NO: 1 SCALE: 1"=80'



LEGEND:
B-1 TO B-3: SPT Borings Drilled in 6/95

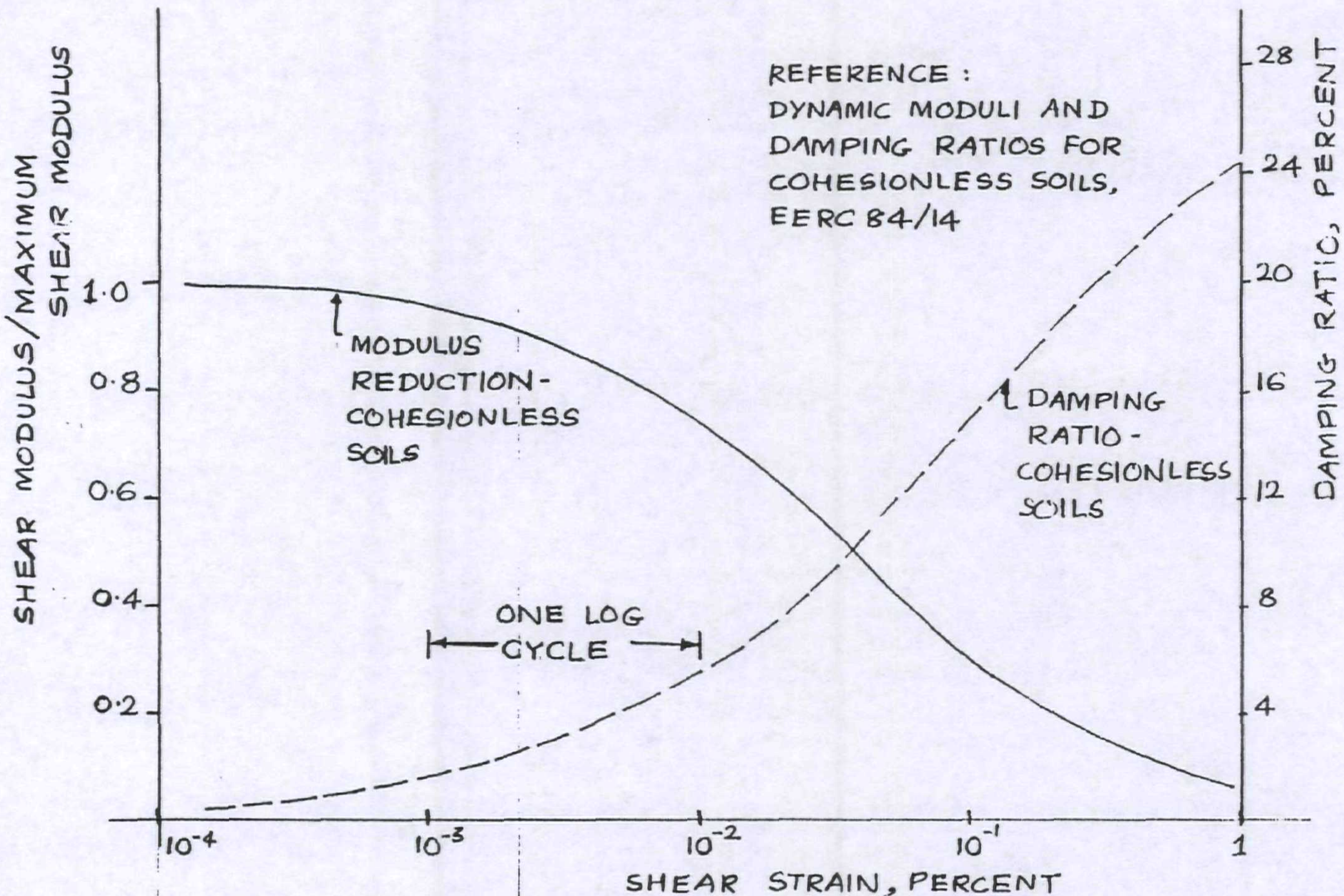
BRAUN INTERTEC	SCHEMATIC SITE PLAN FRED DEVINE DIVING & SALVAGE 6211 N. ENSIGN PORTLAND, OREGON	DRAWN BY: R.F. JOB NO: EAAX-95-0286 DATE: 7-18-95 DRAWING NO: 1 OF 1 FIGURE NO: 2 SCALE: 1"=80'



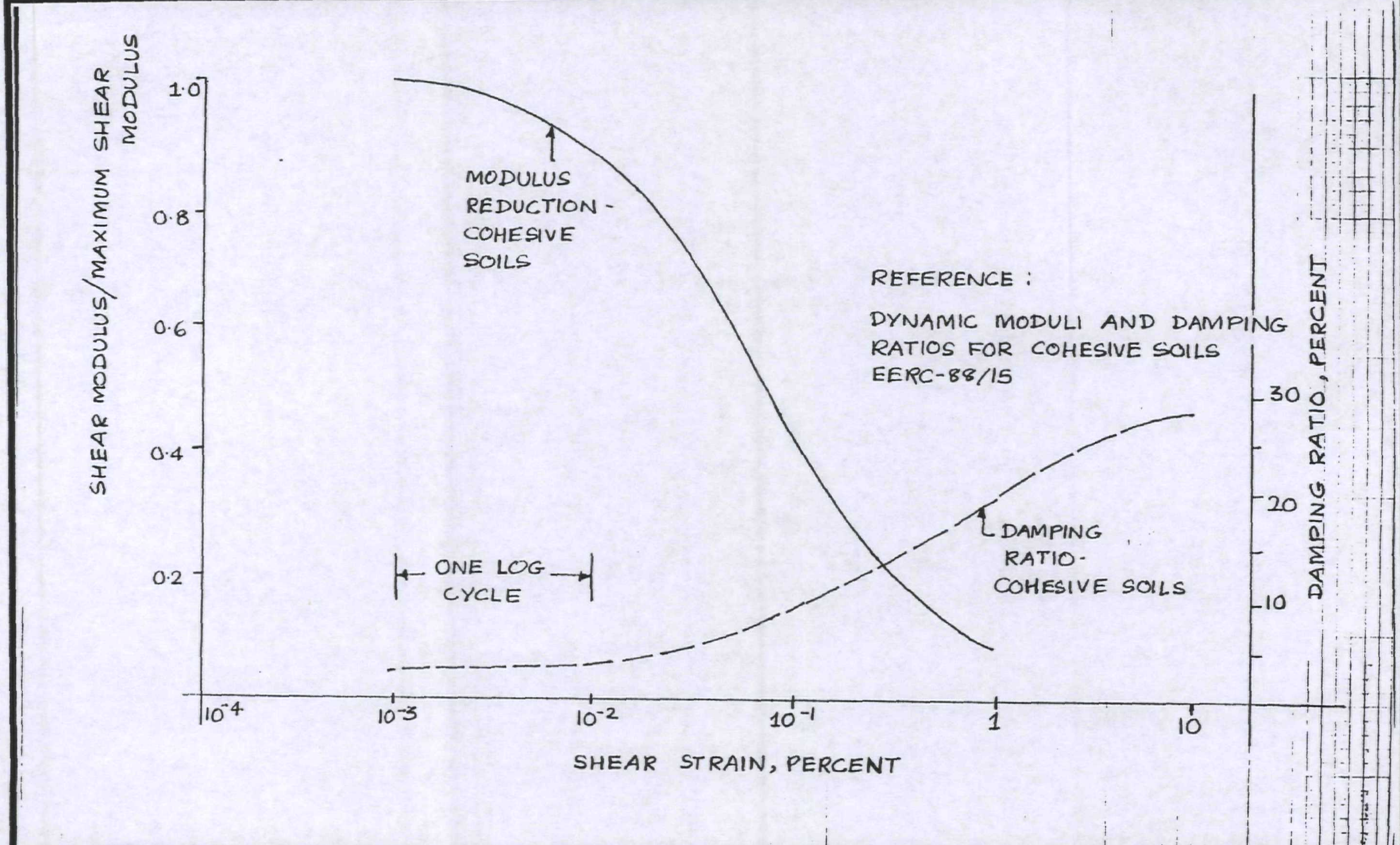
BRAUN INTERTEC

MAXIMUM SHEAR MODULI PROFILE
FRED DEVINE & SALVAGE COMPANY
6211 N. ENSIGN
PORTLAND, OREGON

DRAWN BY: S.A.
JOB NO. EAAX-95-0286
DATE: 7-12-95
DRAWING NO: 1
FIGURE NO: 3
SCALE: AS SHOWN



BRAUN INTERTEC	MODULI REDUCTION & DAMPING RATIOS FRED DEVINE & SALVAGE COMPANY 6211 N. ENSIGN PORTLAND, OREGON	DRAWN BY: S.A. JOB NO. EAAX-95-0286 DATE: 7-12-95 DRAWING NO: 1 FIGURE NO: 4 SCALE: AS SHOWN



BRAUN INTERTEC	MODULI REDUCTION & DAMPING RATIOS FRED DEVINE & SALVAGE COMPANY 6211 N. ENSIGN PORTLAND, OREGON	DRAWN BY: S.A. JOB NO. EAAX-95-0286 DATE:7-12-95 DRAWING NO: 1 FIGURE NO: 5 SCALE: AS SHOWN

Appendix A
SPT Boring Logs

DRILLING COMPANY: BRAUN INTERTEC RIG: SIMCO 2400 DATE: 07-11-95

BORING DIAMETER: 8" DRIVE WEIGHT: 140 lbs. DROP: 30" ELEVATION:

SOIL DESCRIPTION

BORING NO. B-1

DEPTH (FEET)	BAG SAMPLE	DRIVE SAMPLE BLOWS/FOOT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0		6			SM	FILL – SILTY SAND – Brown, loose, moist, trace organics
		8			SP	DREDGED SANDY FILL – SAND – Brown to gray, medium grained, very loose to loose, moist
5		2				
		4				
10		4			SM	FILL – SILTY SAND – Brown, fine grained, very loose, moist
		34			GM/SM	OLD GRAVEL PAVEMENT – Mixture of rocks, sand, and gravel, moist
15		22			SP	SAND – Brown, fine to medium grained, medium dense, moist to very moist
20		12				
25		9			ML & SP	MIXTURE OF SILT & SAND – Brown, fine to medium grained SAND mixed with bluish-gray SILT, loose and soft, very moist to wet
30						

BORING LOG

BRAUN INTERTEC	WAREHOUSE & OFFICE BUILDING EXPANSION 6211 N. ENSIGN PORTLAND, OREGON	BRAUN INTERTEC
PROJECT NUMBER: EAAX-95-0286		Appendix A

DRILLING COMPANY: BRAUN INTERTEC	RIG: CME 75	DATE: 07-11-95
BORING DIAMETER: 8"	DRIVE WEIGHT: 140 lbs. DROP: 30"	Page 2 of 7

SOIL DESCRIPTION

BORING NO. 1 (Continued)

DEPTH (FEET)	BAG SAMPLE	DRIVE SAMPLE BLOWS/FOOT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
30		2			ML & SP	MIXTURE OF SILT & SAND – Brown, fine to medium grained SAND mixed with bluish-gray SILT, loose and soft, very moist to wet
35		2				
40		2			ML	SILT – Bluish-gray, very soft, wet
45		0				Moist below 45 Ft.
50		0				
						Total Depth – 51.5 Ft. Groundwater Encounterd @ 25.0 Ft.
55						
60						

BORING LOG

BRAUN INTERTEC	WAREHOUSE & OFFICE BUILDING EXPANSION 6211 N. ENSIGN PORTLAND, OREGON	BRAUN INTERTEC
PROJECT NUMBER: EAAX-95-0286		Appendix A

DRILLING COMPANY: BRAUN INTERTEC		RIG: SIMCO 2400	DATE: 07-11-95
BORING DIAMETER: 8"	DRIVE WEIGHT: 140 lbs.	DROP: 30"	ELEVATION: 136.0

SOIL DESCRIPTION

BORING NO. B-2

DEPTH (FEET)	BAG SAMPLE	DRIVE SAMPLE BLOWS/FOOT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0		8			SP	DREDGED SANDY FILL - SAND - Brown to fine to medium grained, loose to medium dense, moist
		14				
5		11				
		15				
10		7				
		4				Very loose @ 12.5 Ft - 14.0 Ft.
15		5				Very moist below 15.0 Ft.
						Total Depth - 16.5 Ft. No Groundwater Encountered
20						
25						
30						

BORING LOG

BRAUN INTERTEC	WAREHOUSE & OFFICE BUILDING EXPANSION 6211 N. ENSIGN PORTLAND, OREGON	BRAUN INTERTEC
PROJECT NUMBER: EAAX-95-0286		Appendix A

DRILLING COMPANY: BRAUN INTERTEC   RIG: SIMCO 2400   DATE: 07-11-95

BORING DIAMETER: 8"   DRIVE WEIGHT: 140 lbs.   DROP: 30"   ELEVATION: 136.0

DEPTH (FEET)	BAG SAMPLE	DRIVE SAMPLE BLOWS/FOOT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION BORING NO. B-3
0		9			SP	Top 3 inch – Crushed ROCK surface DREDGED SANDY FILL – SAND – Brown to fine to medium grained, loose to medium dense, moist
		6				
5		12				
		11				
10		3				
		5				Very loose to loose below 10 Ft.
15		3				
						Total Depth – 16.5 Ft. No Groundwater Encountered
20						
25						
30						

BORING LOG

BRAUN INTERTEC	WAREHOUSE & OFFICE BUILDING EXPANSION 6211 N. ENSIGN PORTLAND, OREGON	BRAUN INTERTEC
PROJECT NUMBER: EAAX-95-0286		Appendix A